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ORDINARY MEETING
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Design and construction of Sydney Tower

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Alex Wargon graduated from the Israel Institute of Technology and subsequently gained his MSc from Harvard University. After working with various consulting firms, he entered private practice in 1961 and, in 1963, merged with Robert Chapman to form the present firm.

As a principal of the firm, he has, in the past 20 years, been in responsible charge of the design of numerous projects throughout the Commonwealth of Australia, New Zealand, and other countries. Mr Wargon was involved in research into the design of tall buildings and, in particular, the response of structures to wind. He is the author of a number of published papers and has lectured at the Universities of New South Wales and Sydney.

Synopsis

The paper describes the evolution of Sydney Tower, introduces some examples of analyses involved in the study of its static and dynamic response, and finally presents some details of construction procedures. The structure of Sydney Tower is possibly unique in the sense that, having been 'tailored' to suit a particular set of circumstances and requirements, it offers a number of unconventional characteristics that make it appear distinctly different from other world-renowned towers. It can be characterised as a free-standing, cantilevered, post-tensioned, guyed steel structure.

Introduction

The famous Eiffel Tower, a centre of controversy during its erection for the Paris International Exposition of 1889, emerged as a major attraction and a highly profitable enterprise. The original intentions for ultimate demolition of the structure were abandoned and the Eiffel Tower, a renowned landmark, became the forerunner of many similar structures throughout the world.

The advent of television and telecommunications gave impetus to the development of towers, particularly throughout Europe. Their locations were governed primarily by specific technical requirements and in many instances the profitable tourist facilities, such as observation platforms and restaurants, were treated as an afterthought, resulting in less than adequate provisions to serve this function.

During the past two decades, a number of towers have sprung up throughout the world, designed specifically to serve tourism, with the added facility of catering for telecommunication. Sydney Tower is an example of such a tourist facility, located in the retail heart of the city, with its captive customer market, a magnificence of scenery and a congenial 'year round' climate; it is a recognised landmark which, together with the Opera House and Harbour Bridge, identifies Sydney on the world map.

Evolution of a structure

The advantages and disadvantages attaching to a host of existing structures and to a wide variety of available structural materials were carefully considered in a first appreciation of the problem, and it soon became apparent that, since the proposed tower was to be constructed on the top of a very large and massive concrete building, a strong argument could be advanced for a relatively light and slender structure. Secondly, evaluation of means of vertical transportation established a minimum shaft of 6·6 m diameter to house the necessary lifts, stairs, and ducts. Such a slender shaft-column required a system of guying to the massive base building, and an array of straight cables offered the simplest solution to the problem. However, in order to improve the torsional rigidity of the structure, it was decided to engage two opposing families of straight, inclined cables, starting from the base, tangential to the shaft and extending to the underside of the turret. Such an array of cables generates a geometrical shape known as a one sheet hyperboloid of revolution. Thus the basic criteria for design were established as follows: light structure (hence steel) guyed to the base building with a series of wire cables. (Fig 1.)

Initial analysis of the proposed system involved consideration of a simple cantilever beam with a varying moment of inertia and section moduli of the gross section of shaft and cables. It is interesting that this somewhat primitive first approach produced surprisingly accurate results, which were later confirmed by detailed frame analysis.

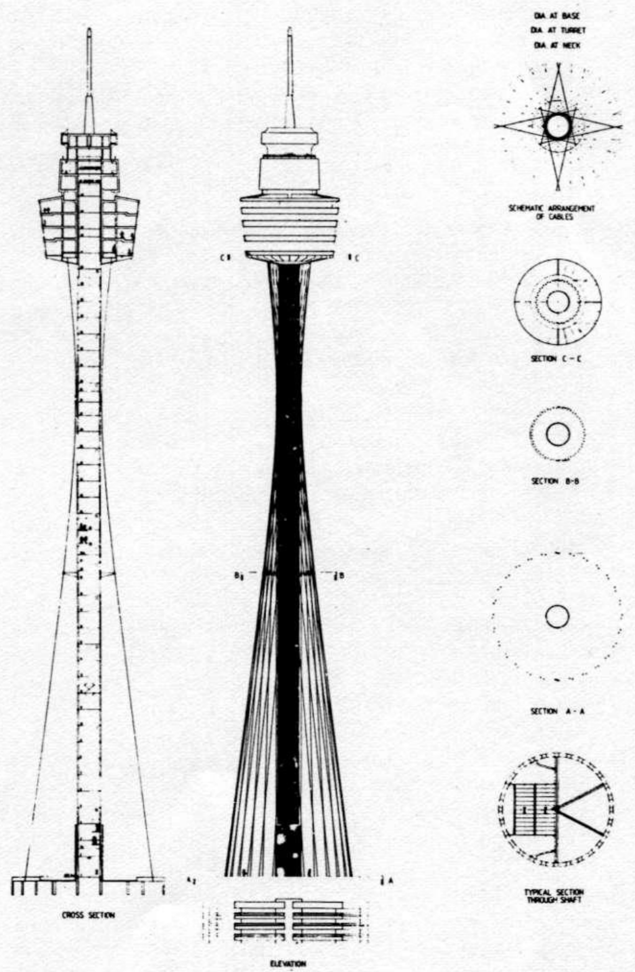


Fig 1. Elevation and cross-section

During the initial design stage, it became apparent that satisfying a major consideration of limitating deflection will result in increasing the size of cables well in excess of that required for strength, thus substantially increasing the factor of safety of the structure.

Preliminary design was based on quasi-static wind loading which was in excess of the requirements of the then current interim Code. This loading was later identified as representing a 4000-year return period wind. This preliminary design work was assisted by the construction and testing of a structural model, which work was conducted at the Department of Architectural Science of the University of Sydney, under the supervision of Prof. J. Gero. (Fig 2.) On the basis of the results of the tests, the various structural elements were confirmed and finally located, including the 'neck' and the intermediate anchorage ring (IRA). The architectural planning of the turret completed the process of evolution of the tower. (Fig 1.)

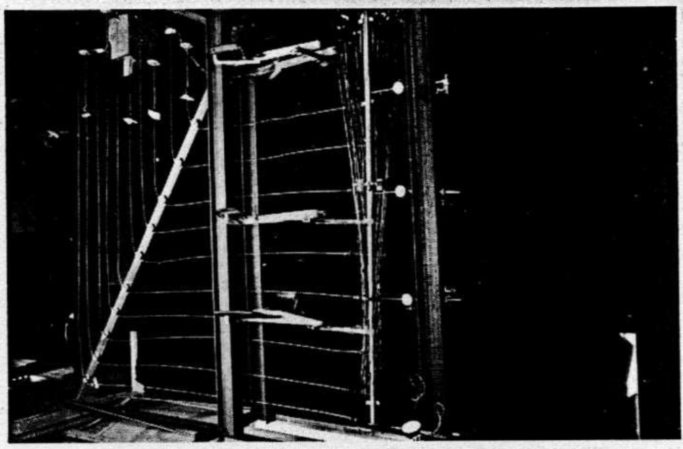


Fig 2. Testing of a structural model at the University of Sydney

Design load and performance criteria

From the point of view of gravitational loads, the building was treated in a conventional manner and hence the live load, which complied with the SAA Code requirements, was superimposed onto the actual weight of the various elements of the tower. However, the other design parameters, such as wind and seismic loads, being dynamic in nature, demanded far greater attention and a much more detailed consideration. Both seismic loads and wind loads, by virtue of their horizontal application, result in similar responses and usually only one of the two governs the design. Sydney Tower, being designed basically for response to maximum wind condition, is inherently capable of withstanding considerable seismic loadings, such as those that could occur in what is known as a 'seismic belt'.

At the commencement of the design stage, the then current Int. 350 Loading Code was based on a rather primitive understanding of the nature of wind loads, and therefore it became essential to establish design criteria that were more appropriate for such a significant structure, particularly since it was recognised that the behaviour of the tower depends greatly on its response to wind. Therefore, detailed information on wind velocity was sought from the Bureau of Meteorology, particularly from observations at both Observatory Hill in Sydney and Mascot Airport. Auxiliary information was also obtained from records of wind velocity measured from balloon observation.

During this time (1969) Prof. A. G. Davenport and his group at the University of Western Ontario were engaged in preparing the Canadian National Building Code which involved a rather novel approach to the appreciation of wind loads. The information gathered in Sydney was subsequently sent to Canada on magnetic tape and, for the first time in Australia, design parameters were adopted for the design of the structure, relating gradient hourly mean wind velocity to that of return period.

The resulting relationship for winds at an assumed gradient height of 360 m was as shown in Fig 3.

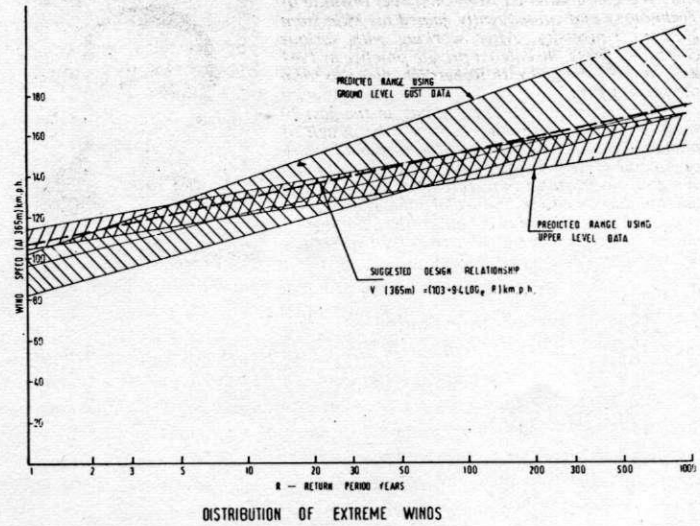


Fig 3. Distribution of extreme winds in Sydney

It is of interest to note that, since 1969, the subsequently formulated Australian Code adopted a similar approach to the design parameters for wind.

The formulation of design performance criteria also necessitated a departure from the conventional approach, with which the practising structural designer is familiar. Naturally, the maximum static and dynamic stresses resulting from gravity load and the response to wind occurring on a return period of 1000 years were to be kept at a permissible level. In addition and in this regard, the magnitude of fatigue stresses was considered.

However, it was appreciated that the conventional and somewhat arbitrary limitation on deformations was not applicable to a structure of this nature. The issue became to ensure that the movement is such as not to cause anxiety or discomfort to the occupants of the tower, or conversely the limitation on deformation and oscillation was designed to ensure comfort of the occupants of the tower. More specifically, it was accepted to limit infrequent acceleration to approximately 1·5%G. It is understood that at this level of acceleration, and especially if the movement is of low frequency, about 90% of the population will just perceive the motion.

Design

The base structure is a multistorey reinforced concrete building containing 25 000 m^3 of concrete and endowed with extensive shear resisting elements in the form of concrete shear walls in various locations. (Fig 4.) In order to ensure participation of this mass of concrete in the process of stabilising the tower structure, 120 no. 60 ton cables were incorporated in concrete walls and columns and post-tensioned after construction of the base building. The main concrete building is surmounted by a rigid concrete box mechanical plant room, 8 m high and 42 m × 42 m in plan. This box is provided with a concrete ring 1·3 m deep and 10 m wide within which both the vertical stressing cables and the tower cables anchor and terminate. Thus the ring beam constitutes a transition structure which effectively couples the tower tension cable with the concrete base building.

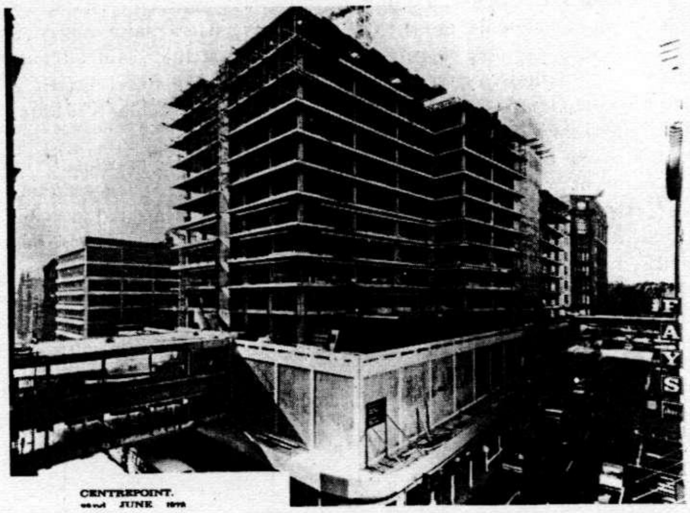


Fig 4. Centrepoint base building during construction

The central shaft is fabricated from 56 no.—250 mm × 250 mm universal columns boxed in pairs and rolled from AUSTEN 50, a 350 MPa weathering steel, produced by Broken Hill Pty. Ltd. Design of the central shaft incorporates the necessary bracing to ensure that the stability of individual boxes exceeded the global stability of the shaft, which is guyed at the neck, located some 150 m above the roof of the concrete building. Design of the stiffening rings and their location is based on consideration of virtual work associated with the various possible modes of buckling. The shaft is prefabricated in 4·8 m-high sections which include stairs, bracing, pipes, and other services.

At a level of about 100 m above the top of concrete building an intermediate anchorage ring (IAR) is located. This structure fulfills several functions; it simplifies the erection procedure, increases stiffness, and reduces the unbraced length of the central column, thus improving the global stability of the structure.

The turret is constructed of 14 fully-welded steel frames, protected by means of fire-proofing spray. The steel frame rests on a boxed base constructed of 25 mm plates and 28 outriggers which radiate to the perimeter. (Fig 5.)

The cables are constructed in three segments. The two lower segments comprise 235 no. 7 mm wires bunched together and strapped at intervals, while the upper segment comprises 55 no. wires only, since it contributes little to the stiffness of the tower. All wires are heavily galvanised.

Static and dynamic analysis

It is of interest to note that, while the detailed static and dynamic analyses involved the utilisation of the most up-to-date electronic devices in Australia, USA, Great Britain, and Norway, these analyses confirmed simply that the structure will behave statically as a simple cantilever beam or a guyed mast. Dynamically, the structure resembles a system with two degrees of freedom, in which vibration is inhibited by the movement of an auxiliary mass. (Fig 6.)

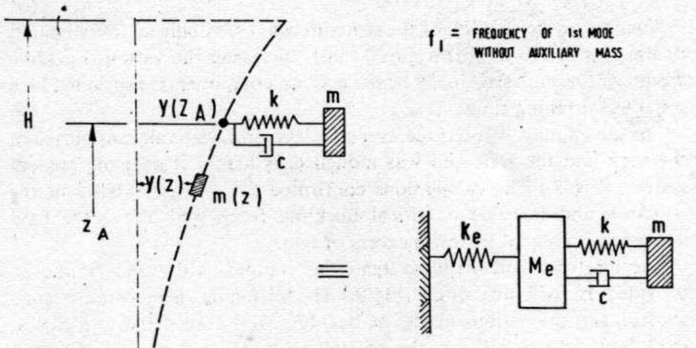
The initial design work involved some basic analyses of the structure including:

(a) Analysis of a cantilever beam with varying moment of inertia. The beam comprises a shaft and 56 cables of constant area of steel and varying position which is a function of the geometry of hyperboloid of revolution. These tedious calculations, which included the calculations of wind-induced stresses and deformations, were assisted by the application of a simple in-house computer program.
(b) Analysis of a guyed mast structure which assumed linear behaviour of prestressed cables.
(c) Determination of natural frequencies.

On the basis of these analyses the structural components of the tower were determined in some details, to allow a more accurate analysis of the global frame, which included:

(a) a 3-dimensional STRUDL-frame analysis of one-quarter of the tower;
(b) a 3-dimensional GASP analysis of one-quarter of the tower;
(c) later, a 3-dimensional GASP analysis of a revised design which incorporated a reduced Young's modulus (E) for the cables and the final, increased size of turret.

The turret structure was analysed using 2-dimensional frame 2 analysis, where the turret base was modelled as a flexible foundation.

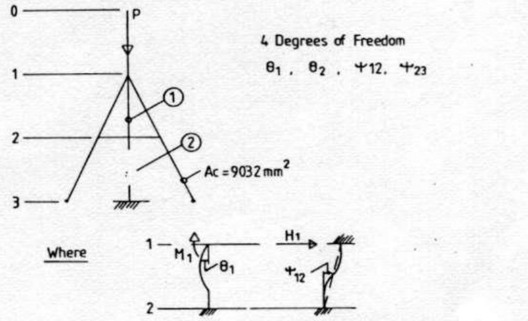


$$M_e = \frac{\int_0^H m(x)\, y^2(z)\, dz}{y^2(Z_A)}$$

$$K_e = (2\pi f_1)^2 M_e$$

EQUIVALENT TWO-DEG.-OF FREEDOM SYSTEM FOR TOWER WITH AN AUXILIARY MASS DAMPER.

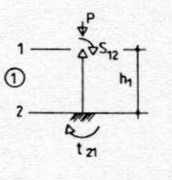
Fig 6. Equivalent two-degree-of-freedom dynamic system



We Find	θ_1	θ_2	ψ_{12}	ψ_{23}	
M_{12}	+12·89	-12·89.	0	0	
H_1	0	0·8379	-0·8379	0	
M_{21}	-12·89	+55·94	-43·01	0	(1)
M_{23}	0	+32·26	0	+43·01	
H_2	0	-0·4821	0	-0·8379	

Units 10^3 MN , 10^3 MNm

M and H are found in terms of prestressing P and element stiffnesses



eg. for member 1

$$S_{12} = S_{21} = \frac{EI}{h_1}\,\frac{\lambda_1(\sin\lambda_1 - \lambda_1\cos\lambda_1)}{2(1-\cos\lambda_1) - \lambda_1\sin\lambda_1}$$

$$t_{12} = t_{21} = \frac{EI}{h_1}\,\frac{\lambda_1(\lambda_1 - \sin\lambda_1)}{2(1-\cos\lambda_1) - \lambda_1\sin\lambda_1}$$

Where:

$\frac{EI}{h_1} = 1342 \times 10^3$ MNm ; $\lambda_1 = h_1\sqrt{\frac{P}{EI}}$

Substituting in (1) with $\lambda_1 = \lambda_2$
for determinant = 0
we obtain $\lambda_1 = 3 \cdot 1514$
$\longrightarrow$ $P_{cr} = 212$ MN. ie. $(l/r)_{cr} = 21 \cdot 7$

Fig 7. Global stability analysis

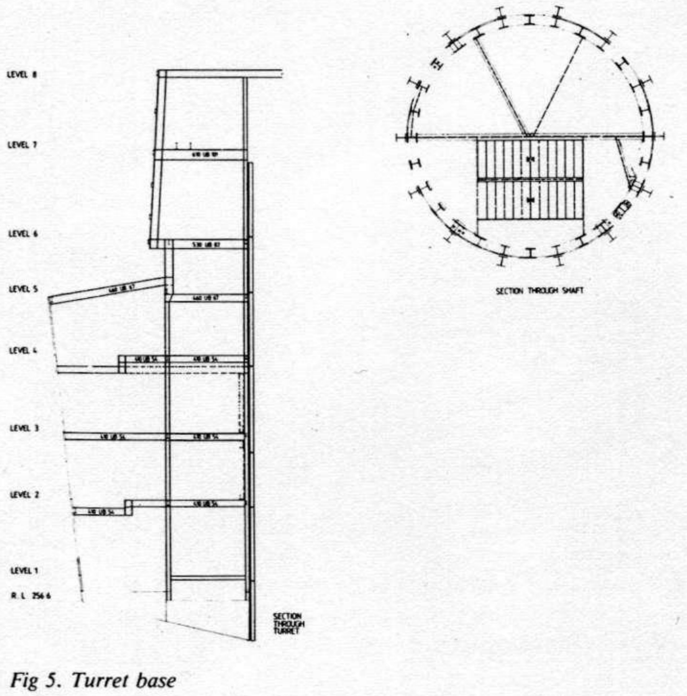


Fig 5. Turret base

Dynamic analyses included:

(a) ASAS—Dynamic programme used to established eigenvalues and frequencies of the system.
(b) The structure was modelled as a two degree of freedom system which included the tuned mass damper and was analysed using an in-house computer program. (Fig 6.) The system was subjected to a random wind gusting, and its responses in terms of deflections and accelerations were studied for varying characteristics of shock absorbers.
(c) A 3-dimensional GASP programme was used for the analysis of the structure when subjected to the historiogram of the El-Centro 1940 earthquake.

Analysis of the stability of the structure was essentially an investigation of the overall stability of a guyed mast, including the various buckling modes of the shaft and its 28 boxed column components, restrained by a series of stiffening rings.

The guyed mast was treated as a cantilever mast laterally supported at the neck and the IAR and was modelled as a four degree of freedom system. (Fig 7.) The calculations confirmed the overall stability of the structure and revealed a critical buckling force well above the load applied, with a load factor in excess of two.

The shaft components were treated as columns with elastic supports, provided by stiffening rings. (Fig 8.) The stiffening rings were designed accordingly, the various modes of buckling were considered, the critical mode was determined. For the critical mode the critical load offered a load factor of about five. For the purpose of the analysis an in-house interactive program was developed, using a computer bureau matrix solution program as a subroutine.

In addition to these global or macroscopic analyses, a series of microscopic analyses was conducted in areas of potential stress concentrations. For example, the detail of the bolted shaft connections involved a series of finite element studies, the purpose of which was to establish the most favourable location of bolts. A full-scale mock-up of the joint was tested to destruction in order to check the tolerance of the steel to local stress concentration. Similarly, a finite element analysis was conducted at the cable anchorage area of the intermediate anchorage ring which effectively pointed to areas requiring stiffening.

During construction, a series of in-house programs was developed in connection with the geometry of the cables. These involved the calculation of the 'draw-in' of cables, which recognises the initial non-linear relationship of stress and strain, and calculation of the geometry during erection to minimise interference with cables already in place. (Fig 9.)

The stressing and final tuning of cables was conducted at night and during still weather. However, since the tower is rather responsive to wind loads, compensating cable forces were calculated for winds of varying intensities, blowing from various directions, and this information was logged together with anemometer readings to provide corrected values of stressing.

It is apparent from the above description of some of the analytical work conducted that, while the structure represents a rather simple and basic concept, extensive probing in depth was necessary in order to study the behaviour of the tower in toto and the various structural elements in particular.

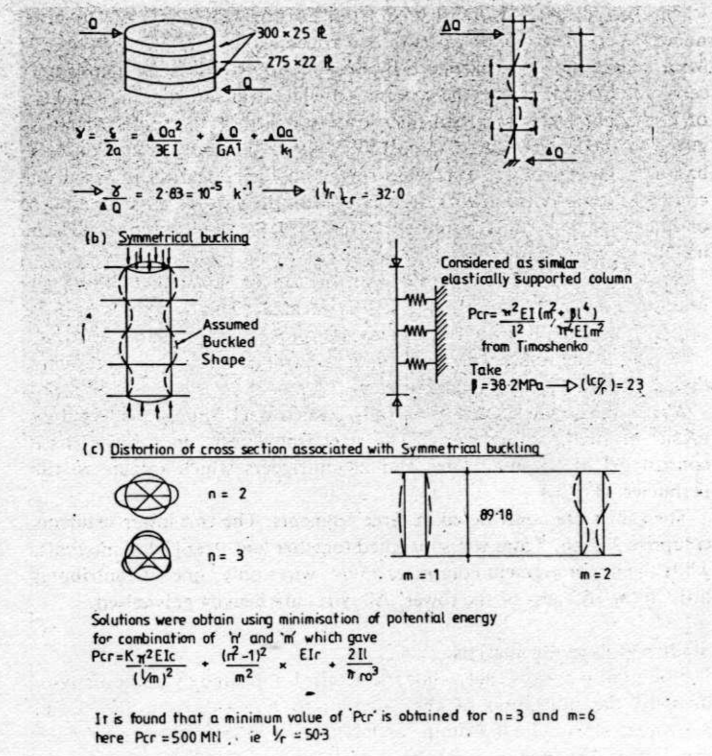


Fig 8. Local stability of steel core

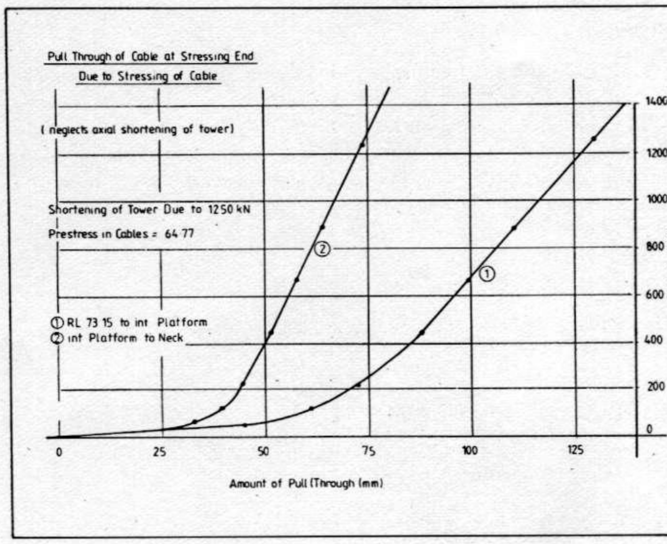


Fig 9. 'Draw in' of cables

Fabrication and testing of the steel structure

The construction of the tower was planned around a principle of maximum prefabrication. In fact, while only some welding was permitted on site prior to erection, welding was totally prohibited in the erected structure.

Fabrication involved the welding of 350 MPa weathering steel, which material was used extensively for the first time in Australia. The design of welding for the project has been the subject of a separate paper presented at a previous conference (Corderoy & Wargon). However, the basic approach to the design of welding, fabrication, testing and inspection procedures deserves a mention in this paper. Of particular interest are the difficulties encountered (and the solutions evolved) in the welding of highly restrained, heavy plate sections, formed into a configuration that is susceptible to lamellar tearing.

The shaft is composed of 56 no. UC sections continuously seam welded along the flange toes, to form 28 no. 250 deep × 500 wide boxed members. The 6·8 m diameter shaft was fabricated in 4·8 m high elements, prewelded in arc segments of 1/7th of the circumference. (Fig 10.) The segments were manufactured in the Johns & Waygoods' plant in

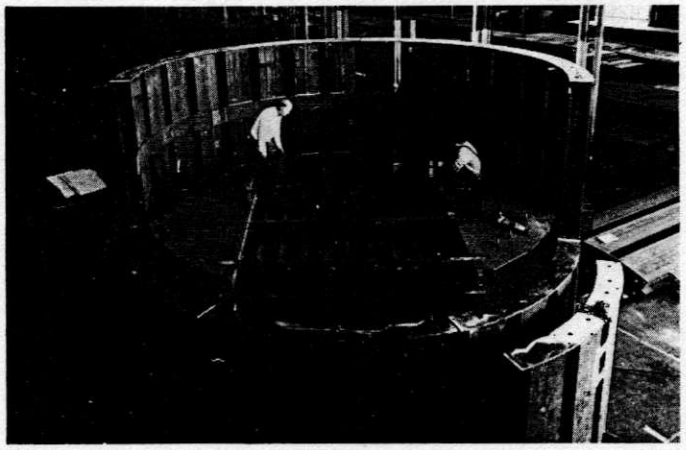


Fig 10. Assembly of shaft elements

Melbourne, transported to the site, and welded into a complete 6·8 m dia. × 4·8 m-high barrel, on top of the concrete base building prior to erection. The prefabricated segments were machined to a tolerance of $\pm$ 0·25 mm on the cap and baseplates and trial assembled prior to the delivery to site. This procedure resulted in a remarkable degree of contact accuracy between bolted shaft barrels, which resisted the insertion of a 0·1 mm feeler gauge.

The turret base comprised an annular box some 16 m in diameter to which 28 outriggers were attached. The 16 m diameter base so formed was prefabricated in 14 segments and assembled and welded on site using the MIG/Co_2 process. (Fig 11.)

The intermediate anchorage ring (IAR) which is a boxed annulus 16 m in diameter was also designed to be constructed in 14 segments. However, the fabricators decided to construct this element in seven segments. (Fig 12.) During erection, the IAR was to be subjected to high stresses, and one of its most important elements is a vertical ring plate adjacent to the point of cable anchorage which, together with the cable tray and its stiffeners, resulted in a highly restrained cruciform joint. The viability of the joint (and the associated welding procedures) was established on a full-scale test section fabricated using the MIG/Co_2 welding process and nickel bearing E5518C1 electrode capping. The test piece was ultrasonically examined for any tears and cracks and then sectioned for extensive mechanical testing. The test joint was cut from plates representative of approximately half of the material used in the IAR which was characterised by a low sulphur (0·017%) content. These tests suggested that the cruciform joints could be constructed satisfactorily.

However, during fabrication, many difficulties were encountered, particularly in relation to lamellar tearing of those plates that were rolled from material having higher sulphur content (0·035%). In order to satisfactorily complete the shop fabrication, it became necessary to apply several techniques developed in connection with welding of joints in material susceptible to lamellar tearing, such as 'buttering' of joints with several layers of low strength E4118 electrodes, replacement of plate with weld metal, replacement of some of the ring plates by special cerium-treated WR 350 2Lo material, some revision of joint details, and extensive gauging in order to stop propagation of tear.

All site welds were subjected to 100 % weld procedure qualification testing and finally were 100 % ultrasonically tested. It is interesting to note that only 0·01 % of length of site weld required repairs.

The most highly stressed element of the structure is the neck connector, the function of which is to provide means of positively anchoring pairs of cables to the shaft at a height some 150 m above the base building. (Fig. 13.) The connectors are fabricated from quenched and tempered 700 MPa, yield stress steel, and comprise: a lower forging, 150 mm thick, an upper forging, 100 mm thick, welded via stress flow transition to a 38 mm-thick sideplate, rolled from ASTMA 542—steel. The fabricator developed a welding procedure for butt welds using E7718-6 electrodes. The connectors were radiographed for compliance with AWS—Bridge Specifications, stress relieved, and finally grit-blasted and hot dip galvanised.

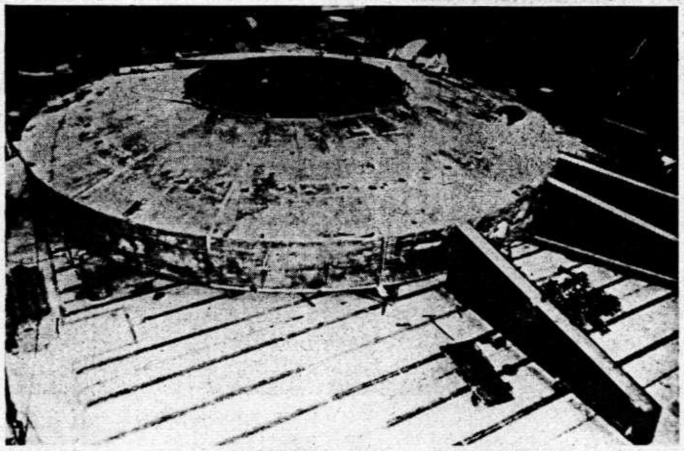


Fig 11. Turret base during construction

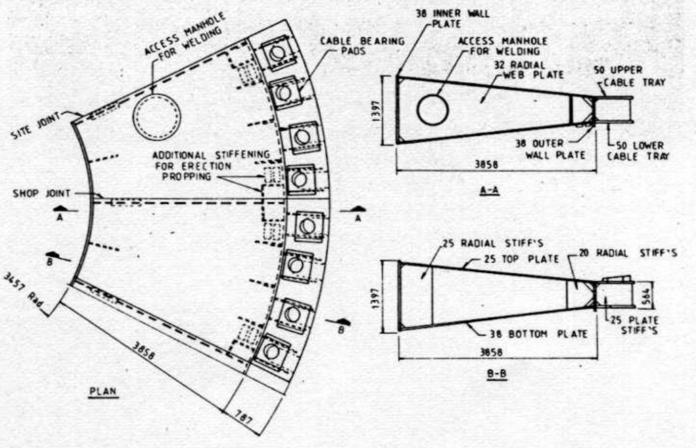


Fig 12. Prefabricated segment of intermediate anchorage ring

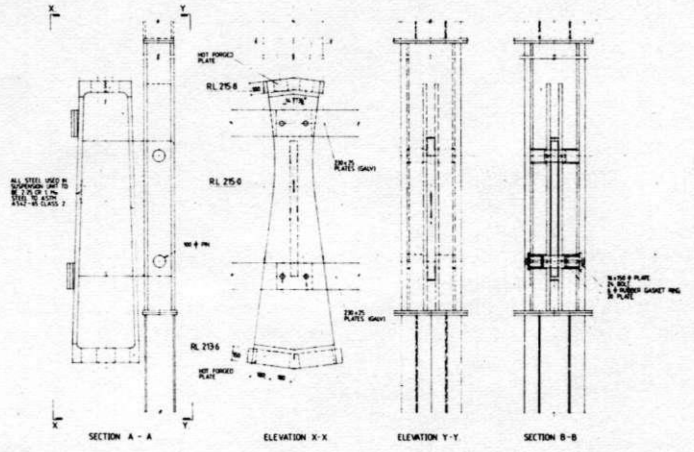


Fig 13. Details of neck connector

The cables

The cables constitute an essential element of the structure. The BBR parallel wire cable system provided maximum stiffness and positive, fatigue-proof type of anchorage, i.e. the BBR DINA anchorage.

The system of guying is made up from two families of cables, one extending right to left and the other left to right, to achieve greater torsional rigidity. The cables are constructed in three segments; the first (about 91 m in length) extends from concrete base to the IAR; the second (some 50 m long) extends from the IAR to the neck; and the third (41 m in length) extends from the neck to the underside of the turret. The total weight of the cables is approximately 700 t.

There are 56 cables, each comprising 235-7 mm dia. wires, bunched together, strapped and coiled for ease of handling during erection. (The cables were manufactured exclusively for this project by Australian Wire Industries, Newcastle, New South Wales, in a joint venture with BBR Australia, Melbourne.)

The nominal tensile strength of the wire was specified to be 1158 to 1313 MPa with a range of 108 MPa above nominal maximum, i.e. a total range of 1158 to 1421 MPa. The wire was hot dipped galvanised, prior to the assembly of cables with a zinc coating complying with BS 443:1969 having a minimum coating weight of 290 g/ m^2.

Each wire was anchored by a cold formed tulip-shaped button head specially developed by Prof. F. Leonhardt for the Bureau BBR Ltd., Zurich, for applications requiring high fatigue resistance.

The cables were manufactured by BBR Australia in their Melbourne factory, coiled and transported to the site. Prior to commencing manufacture, a detailed analysis was carried out by BBR Australia to determine optimum cable lengths, taking into consideration such factors as temperature effects, elastic shortening of the steel shaft (due to prestress), and construction tolerances.

The cable manufacture involved the setting up of a 100m-long cutting and assembly bench, combing devices to assure parallelity of all wires, and special twisting and reeling equipment to handle fully-assembled cables weighing up to 7t each.

To facilitate the winding of the manufactured cable on a drum without birdcaging, all cables have a built-in twist with a pitch length of 9·0 $\pm$

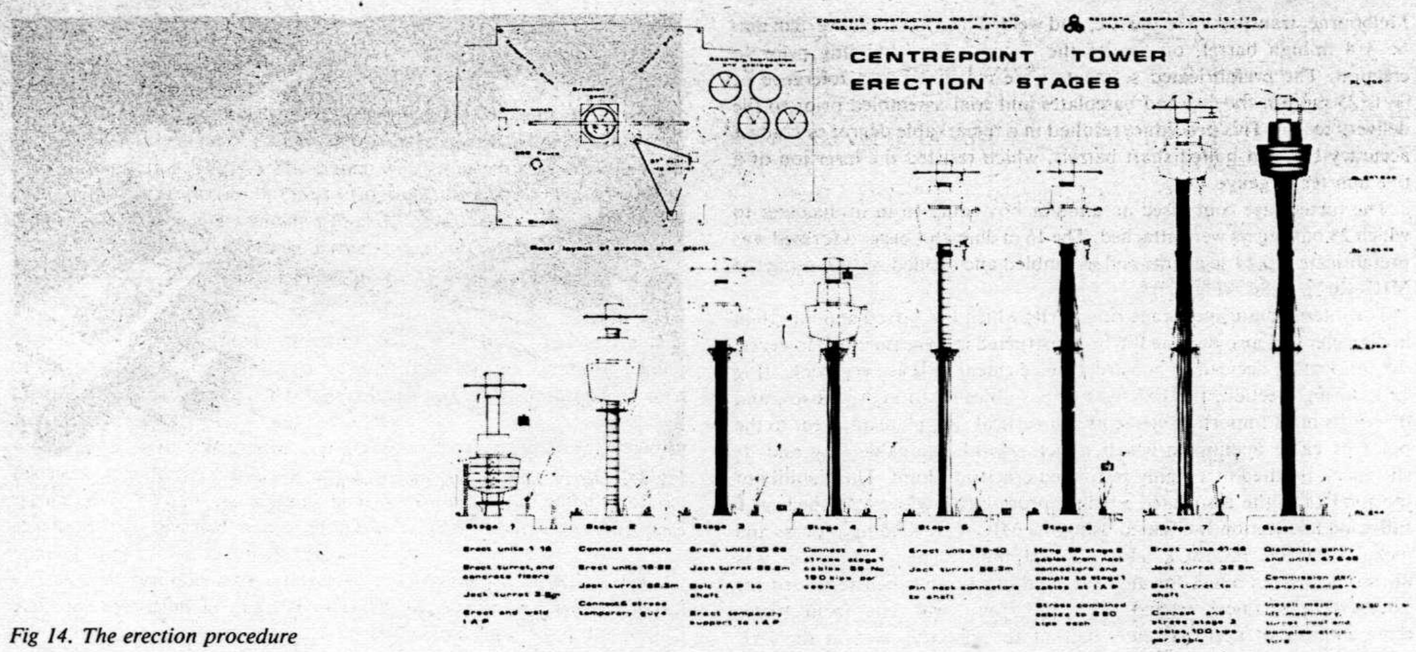


Fig 14. The erection procedure

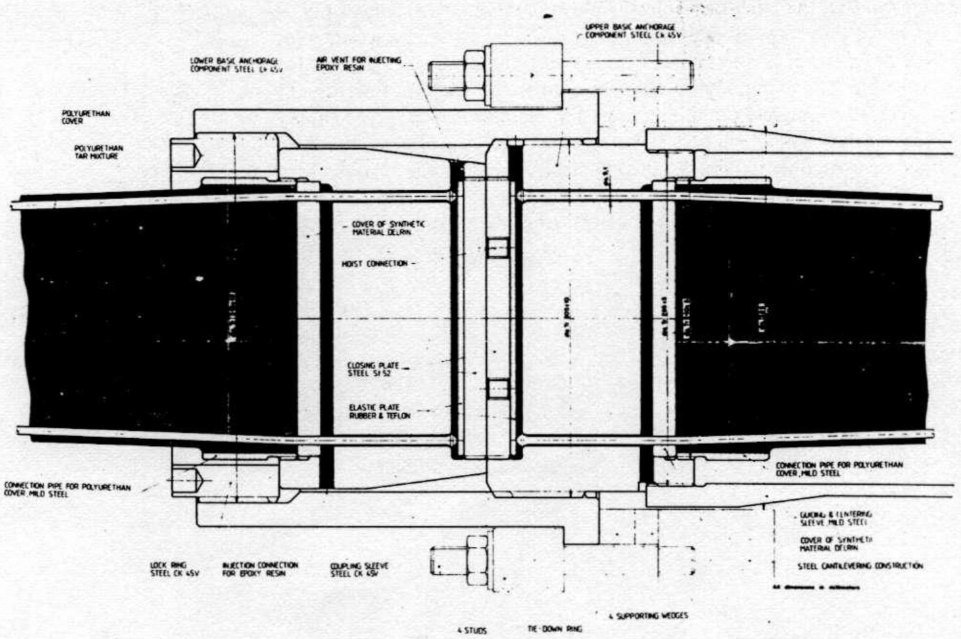


Fig 15. Details of cable coupling

0·3 *M*. Half the total number of cables have a left-hand twist and the other half, a right-hand twist.

The first set of 56 cables to be manufactured and supplied to site during 1977 (albeit the last to be installed) was the 41m-long stage III cables extending from the neck connector to the underside of the turret. These cables, weighing approximately 800 kg each and consisting of 55 no.7 mm dia. galvanised wires, were stored in reels in the turret and travelled with the turret up to the top of the shaft during the erection.

The manufacture of the 91m-long 235 no. 7 mm dia. stage I cables each weighing close to 7 t took place during 1977, followed by the 50m-long 235 7 mm dia. stage II cables which were manufactured during early 1978.

The erection

The method of construction evolved around the principle of maximum prefabrication. The success of this method may be illustrated by the fact that the \$25M structure was erected on the site by a team of 12-20 people, including riggers, welders, labourers, and the supervisory staff.

On completion of the concrete-base building, the prefabricated segments of shaft were transported to the site and placed in a special rig on the roof, welded together, furnished with stairs, lift guides, various hydraulic pipes, etc., to form a single 4·8-m high 6·8 m diameter barrel, which was then lifted into position by a specially constructed, hydraulically rising derrick. (Fig 14.)

On completion of the first 60 m of shaft, construction of the turret commenced. This nine-storeyed, 25 m diameter building, comprising steel frames, concrete decks and spandrels, was built around the shaft. Construction included the erection of a 165 000 litre steel tank built in the form of an anulus, which is suspended on 10m-long cables and connected to the main tower through eight shock absorbers. This suspended water tank forms a tuned mass damper (TMD) which provides the means of mechanical damping.

The intermediate anchorage ring was prefabricated in seven segments and site welded together to form a 'space saucer'. It was installed under the turret and suspended from it, permitting both the turret and the IAR to travel up the shaft together. Erection of the shaft units proceeded until the turret and suspended IAR were jacked to the first parking position at a level of approximately 200 m above the street, thus allowing for the installation of the only temporary guys used during construction. The turret was then hoisted to its second parking position approximately 220 m above street level, thus permitting the intermediate anchorage ring to be bolted to the shaft through some 420-40 mm diameter high tensile bolts and the subsequent erection of the 56 no. stage I cables. At this stage the cables were post-tensioned to a load of 850 kN each, and the erection of additional barrel units proceeded. The turret was lifted to its third parking position, some 270 m above street level. The neck connectors were then constructed, permitting the installation of the stage II cables

which were then coupled with the stage I cables and stressed to a load of 1200 kN each. This was followed by lifting of the turret to its final position, securing it by some 560 no. 40 mm diameter bolts and the installation of the final stage III cables.

The main contractor prepared a detailed programme of the cable installation sequence which allowed for minimum interference by cables still to be installed with either those already installed or, for that matter,the four pairs of temporary guy cables. After overcoming some initial difficulties, the contractor averaged an installation rate of two cables/day. The erection of stage II cables proved somewhat easier, partly because of the reduced weight of these shorter cables. All 28 inner cables were erected first and their lower end tied to the tower shaft. The 28 outer cables were then installed and their lower end coupled to the stage I cables at the IAR level. This was followed by coupling of the 28 inner cables to their corresponding stage I cables at the IAR level. (Fig 15.)

The stage III cables—stored on reels in the turret—were installed at a rate of up to six cables/day. Each cable was unreeled inside the turret and lowered towards the neck connector where the lower anchorage was made fast. The top end of the cable was then secured to its stressing anchorage located at the easily accessible baseplate of the turret. All stressing operations were carried out by BBR Australia, operating with four stressing crews simultaneously, each with its own set of stressing equipment, which included a hydraulic pump, stressing jack, dynamometer, and other accessories. The use of dynamometers permitted the actual load in each cable to be determined with an accuracy of ± 1%. (Fig 16.).

All stressing was carried out at night in order to negate any temperature effects that might have otherwise influenced the cable behaviour. Initially, eight outer cables were stressed to 500 kN, two in each quadrant, in order to clear the first four inner cables stressed. The remaining cables were then stressed, four at a time, two inners and two outers sequentially, working in an anticlockwise direction. After 24 cables (plus the initial eight outers) had been stressed, the temporary guys were released and their lower ends removed from the anchorage.

Basically, all stressing of the stage I cables was carried out in two operations, i.e. all cables stressed to 500 kN and then restressed to 850 kN.

On completion of each of the above stressing operations, each group of four cables stressed simultaneously was over-stressed by a progressively reduced amount in order to achieve an acceptable uniform prestress level in all cables.

Stage II cables, after being coupled in groups of four at the IAR level to the corresponding and detensioned stage I cables, were stressed as a combined cable of a total length of 141 m (91 + 50). This stressing operation was also carried out in two stages, i.e. all 56 coupled cables were stressed to 900 kN during the daytime and then restressed to 1200 kN during three consecutive nights.

Stage III cables comprising 55 no.77 mm dia. galvanised wires were stressed to 500 kN each, in one single operation. Two inner and two corresponding outer cables were stressed simultaneously from stressing points inside the turret.

During February 1981, all cables were checked for existing prestress levels and retensioned to 1200 kN (stage I and II) and 500 kN (stage III).

Testing and research

At the very outset, it became evident that the design of the tower in general and prediction of its behaviour under wind in particular would necessitate extensive research and testing. One of the earlier researches involved work on an architectural model in the aeronautical wind tunnel of Sydney University which was followed by a study of a structural model conducted at the Department of Architectural Science of Sydney University. These studies permitted the determination of a more precise definition of the components of the structure with respect to such matters as the inclusion of the IAR, connection of cables at the neck, and the elimination of tied joints at the nodal intersection points of the cable network. The outcome of these studies led to the planning, design and ultimate construction of an aeroelastic model at the University of Western Ontario. (Fig 17.) Since the boundary layer wind tunnel at this University is about 30 m long, and the model was scaled at 1 in 400, a relatively extensive area around the tower was provided. Turbulence was reproduced by means of a construction of blocks which simulate an average large city building condition with a more detailed block reproduction of the actual existing and proposed buildings within close proximity. It was found that such large adjoining buildings in general

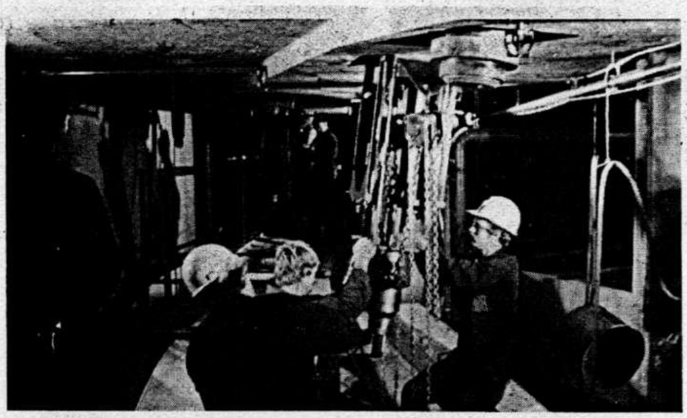


Fig 16. Stressing operation

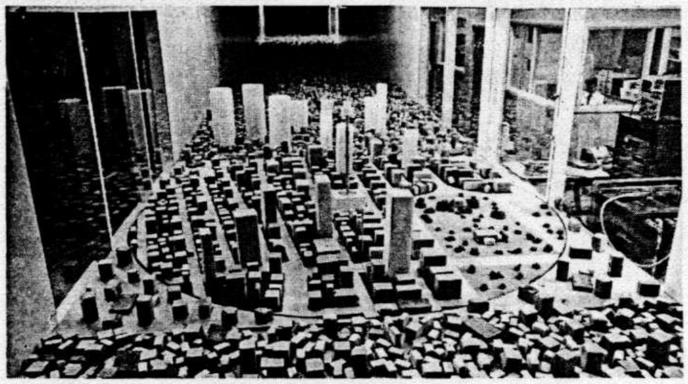


Fig 17. Testing of aeroelastic model at the University of Western Ontario

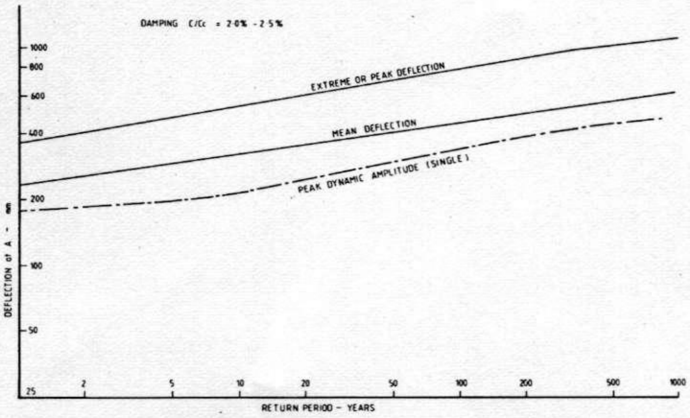


Fig 18. Extreme deflections

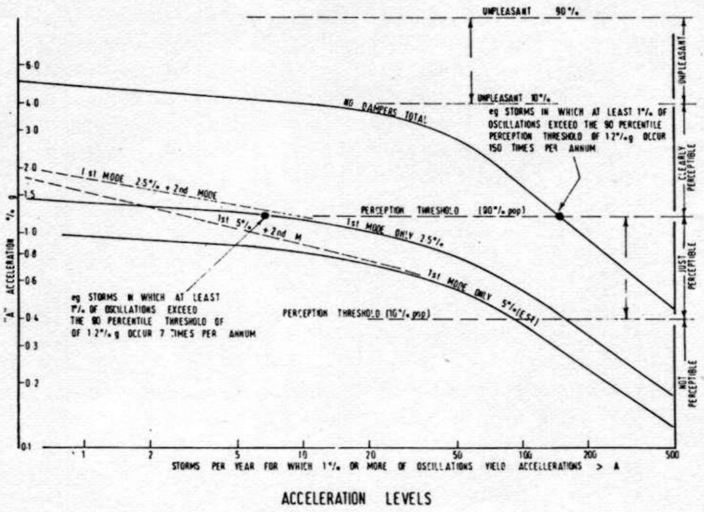


Fig 19. Extreme accelerations

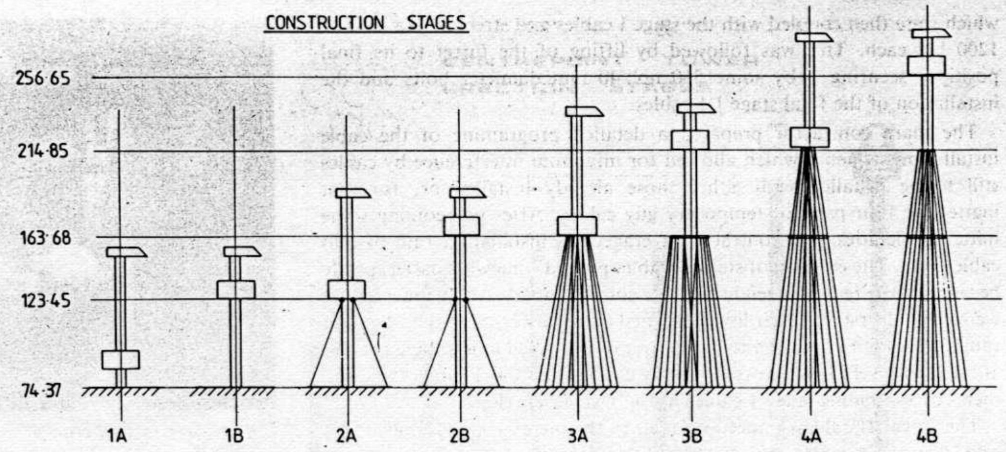


Fig 20. Construction stages investigated in the wind tunnel

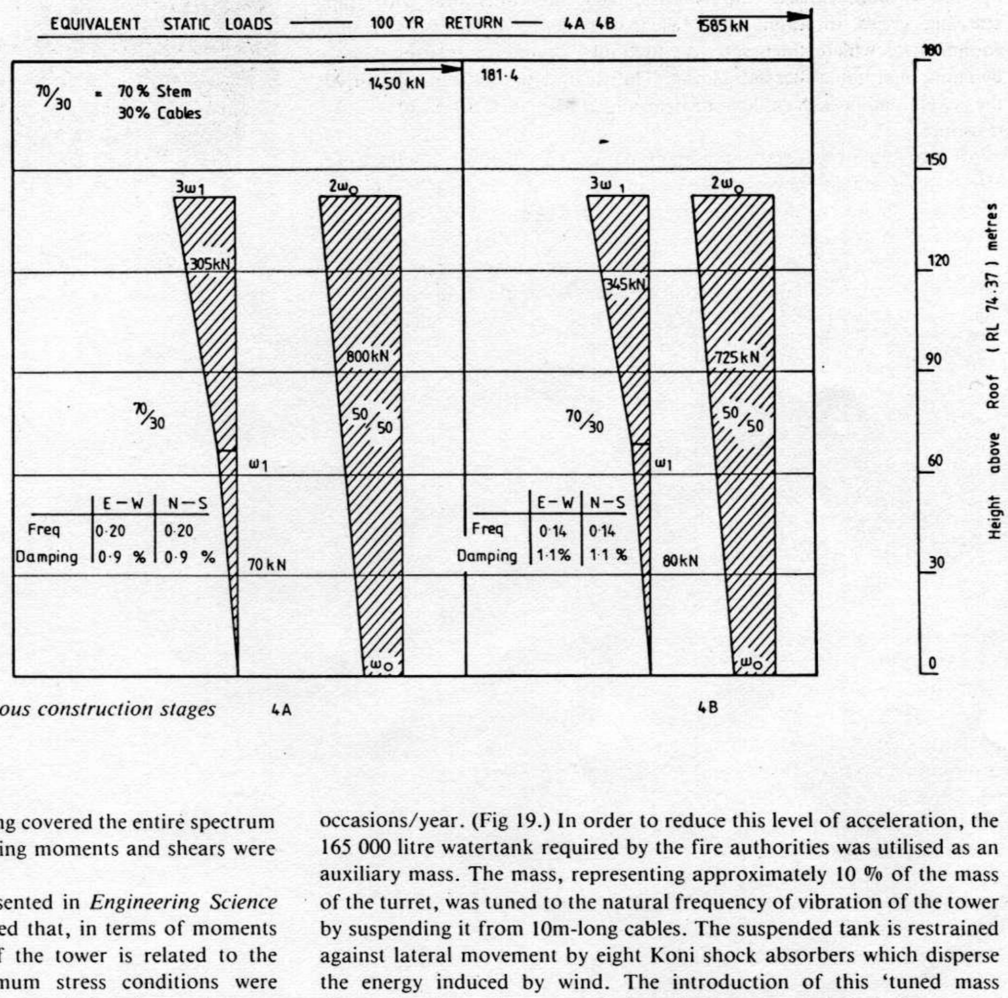


Fig 21. Equivalent static loads for the various construction stages

have a beneficial screening effect. The testing covered the entire spectrum of wind velocities, and accelerations, bending moments and shears were measured. (Fig 18.)

The results of the test, which were presented in *Engineering Science Research Report No. BLWT-1-70,* indicated that, in terms of moments and shears, the most critical response of the tower is related to the fundamental mode of vibration. Maximum stress conditions were associated with the drag loads rather than across-wind motions and even a return period of 1000 years did not produce stresses greater than permissible. Furthermore, with this type of loading, no slacking in cables would occur, since these are prestressed to 1200 kN. However, in terms of acceleration, the studies revealed that the most critical response of the tower to wind gusting is related to the fundamental frequency (0·1 Hz), superimposed on the second mode (0·5 Hz).

The second mode of vibration appeared to be induced by vortex shedding and occurred in the across-wind direction. The peak response in this mode was observed during wind velocities of approximately 35 m/s and involved movements of 50 to 75 mm at the level of the IAR. It was also observed that, in the drag direction, the low structural damping characteristic of a steel structure (of approximately 0·6 % critical) is assisted by aerodynamic damping of the same order. However, since the vibration in the lift direction is not inhibited by aerodynamic damping, the resulting acceleration produced discomfort on some 150 occasions/year. (Fig 19.) In order to reduce this level of acceleration, the 165 000 litre watertank required by the fire authorities was utilised as an auxiliary mass. The mass, representing approximately 10 % of the mass of the turret, was tuned to the natural frequency of vibration of the tower by suspending it from 10m-long cables. The suspended tank is restrained against lateral movement by eight Koni shock absorbers which disperse the energy induced by wind. The introduction of this 'tuned mass damper' reduces the across-wind motion to an extent that, for a wind speed corresponding to a return period of 1 year, the peak acceleration at the turret level is approximately 1·5 %*G* which figure compares favourably with reported acceleration levels in existing buildings of similar heights. It is estimated that, on about five occasions/year, some people would be just able to subjectively perceive movement. The maximum deflections corresponding to a 50-year return period are slightly in excess of 600 mm, of which approximately 375 mm would be due to static deformation with additional 225 mm due to dynamic motion.

Theoretical studies were conducted, also, with respect to the resistance of the structure to fire, leading to acceptance by the authorities of this open steel structure.

In early 1973 an additional investigation was conducted at the boundary layer wind tunnel of Sydney University for the purpose of predicting the response of the structure during the various critical stages

of erection. These stages were identified as being associated with the initial and final dispositions at each of the five parking positions of the turret. (Fig 20.) As the basis of these studies, a 100-year return period wind was selected and the equivalent quasi-static loads were made available for the purpose of designing the structure during erection. In establishing appropriate wind velocities, some difficulties were encountered in selecting the most suitable terrain category and finally it was agreed to adopt an hourly mean wind velocity of 36 m/s at a height of 200 m. It was recognised, also, that such strong winds will, in all probability, originate from the westerly or near-westerly direction. The loading for each stage comprised concentrated loads at the centre of the mass of the turret and distributed loads along the stem and cables. (Fig 21.) The results of this investigation permitted a more accurate design of the temporary guys.

The study was concerned, also, with the expected level of wind-induced vibration as measured by acceleration during the construction, particularly in view of the suggestion that the actual damping capacity of the structure at this stage could be as low as 0·3 % critical. For this level of damping the peak acceleration for a return period of 1 year would have been of the order of up to 8 %G. Since it was recognised that such a level of acceleration would not only impose severe restrictions on the construction programme, but also, perhaps, have undesirable psychological effects on the workforce, it was decided to introduce a system of mechanical dampers, which in effect reduced this acceleration to an acceptable level. In fact, during the entire period of construction, even at times of severe storm, hardly any movement was detectable by members of the workforce.

During construction, and on several occasions/month, the lift compensating and trailing cables were observed to move in a manner that caused some concern. After lengthy consultations, it was decided that the only practical manner of overcoming the problem was to partially enclose the lift shafts. As a result of extensive studies, conducted at the University of Western Ontario, to establish the aerodynamic consequence of such enclosure, it was decided to include an additional tuned mass damper, tuned to the second mode (0·5 Hz). This second TMD comprises a 40t steel ring which is suspended on 1·2m-long rods at the IAR and is connected to it through seven shock absorbers. (Fig 22.)

The tower is presently extensively instrumented with synchronised anemometers, accelerometers, and a series of strain gauges, to allow monitoring of its structural response to wind gusting. This work is being carried out by the Department of Civil Engineering, University of Sydney, under a grant offered by the AMP Society. Since completion, the tower has responded very satisfactorily to buffeting by severe winds which occurred on several occasions registering gusts of up to 45 m/s.

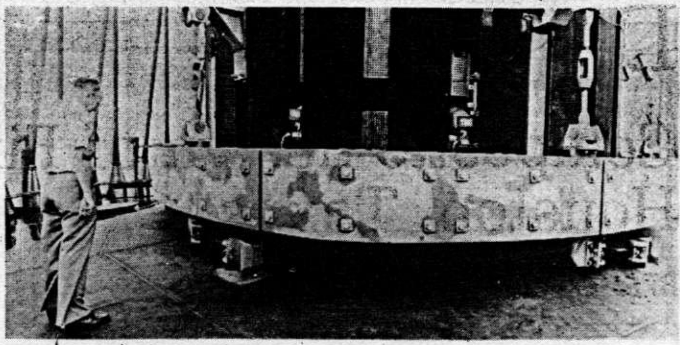


Fig 22. Second mode tuned mass damper

Fig 23. Bird's view of Sydney Tower

Conclusion

Sydney Tower was opened to the public in September 1981 and, in the first year of operation, was visited by some 1·3 M people. In addition to its function as a tourist facility, it is already in use as an important communication link, although this facet of the building has not yet been fully utilised. It is expected that its potential will be fully exploited in the years to come.

The tower is also used as a navigational landmark by both air-and sea-craft and can be seen from a distance of 30 km and more.

Sydney Tower has now become an accepted and integral part of the Sydney scene. (Fig 23.)

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